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Design of the cross-section of a steel composite bridge taking into account the buckling check according to EN 1993-1-5

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Abstract

As part of optimizing a steel composite section to determine whether longitudinal stiffeners could be omitted concerning plate buckling, a buckling tool was developed that implements both the effective width method (EWM) and the reduced stress method (RSM). These two methods are offered in addition to the finite element method in EN 1993-1-5 for buckling analysis. It was found that when using the formula given in EN-1993-1-5 for calculating the critical buckling stress of unstiffened buckling fields, which is independent of the acting stress distribution, the ratio of the elastic critical plate buckling stress to the elastic critical column buckling stress ($\sigma_{cr,p}/\sigma_{cr,c}$) is underestimated if the compressive stress is not uniform. This affects the detection of column buckling behavior. The most economical variant resulting from the optimization, the buckling tool, the influence of shear deformations combined with plate buckling, and the column buckling behavior of unstiffened buckling fields are presented in this paper.

Keywords

Plate buckling, stiffened plate, thrust distortion, column-like behavior

1 Introduction

Plate structures can buckle due to compression and shear stresses in their plane. This stability problem, "plate buckling", plays an important role in steel bridge construction due to the slender components used. The cross-sections of box girder bridges are usually formed with longitudinal stiffeners to counteract the "plate buckling" phenomenon. For the buckling of stiffened plates, it is necessary to distinguish between the failure of individual fields and the failure of the entire field. This makes the check more complex than other resistance checks in a static calculation. As a result, this check is often only carried out at the relevant points of a bridge, which can affect the optimization of the cross-section and economic efficiency. The increasing use of software programs that use numerical solution methods, such as the Finite Element Method (FEM), makes it possible to check component resistances determined by manual calculation formulae using load capacity calculations. In addition, it is possible to extend the buckling analysis to the whole structure by using spreadsheet programs, which provides further opportunities for optimizing the section design. The programming of interfaces for the coupling of all software programs used for static calculation and verification could enable faster verification for the entire structure.

First, the possible verification methods for plate buckling of plate-shaped components in steel structures according to DIN EN 1993-1-5:2019-10 [1] are presented. Subsequently, further issues regarding the buckling verification of stiffened plates, the joint consideration of plate buckling and shear deformations, and the column-like behavior of unstiffened plates are investigated and clarified. Subsequently, automated buckling checks are created by programming several software interfaces. The aim is to shorten the manual, time-consuming verification process by linking the buckling analysis directly to the static calculation of the overall structure and keeping it in a spreadsheet program. Based on the developed verification tools, the structure shown in Figure 1 is calculated and optimized in several stiffening variants. Two decisive variants, one with longitudinally stiffened and one with unstiffened plates, are proposed as possible designs and evaluated for economic efficiency.

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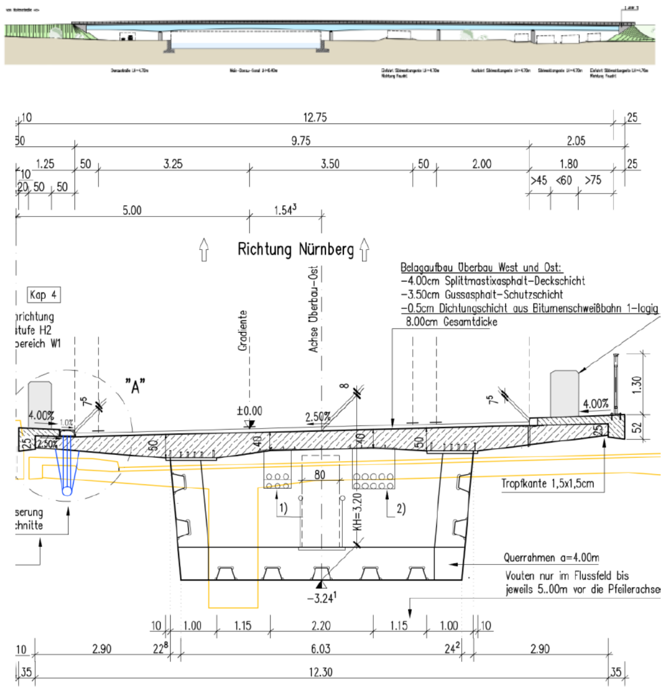

Figure 1 view and cross-section [2]

2 Presentation of the buckling verification methods and discussion of normative approaches

2.1 Current plate buckling verification methods

EN 1993-1-5 [1] provides three possible methods for buckling verification:

- The effective width method (EWM) is described in sections 4-7.
- The reduced stressed method (RSM) according to section 10.
- The Finite Element Method (FEM) in accordance with Annex C.

Compared to RSM, EWM is generally considered the more economical method, as it allows stress redistribution between the individual cross-sectional parts, thus making better use of the cross-sections [3]. However, RSM is fast and comparatively easy to use as it does not require recalculation of cross-sectional values and separate checks for each stress component but rather calculates modified plate slenderness based on the entire stress field. The FEM check consists of modeling individual buckling fields, or even larger components, using so-called shell elements using appropriate software that solves eigenvalue problems numerically. Correctly selecting many parameters, such as mesh density, support conditions, and load application, is very important. These models are then used as pre-deformations for a non-linear load capacity calculation. In Germany, the EWM is currently only approved for unstiffened plates in bridge construction [4].

2.2 Buckling verification methods for stiffened plates

According to RSM, there are two possibilities for buckling verification, which will be explained in more detail below, analogous to their designation in [5] as Method 1 and Method 2. In the case of buckling verification according to method 1, the smallest critical amplifying factor included buckling is determined numerically, regardless of whether it is a buckling of the entire buckling field (i.e., also a buckling of the stiffeners) or only a buckling of partial fields (sub-panel between the stiffeners and edges). The critical amplifying factor can thus be expressed as $\alpha_{cr} = \min(\alpha_{cr,global}, \alpha_{cr,local})$. This smallest factor is used to determine the modified slenderness, which is then used to determine the reduction factors ρ_x, ρ_z taking into account the column-like behavior of the overall field and χ_w.

According to method 2, the verification assumes that, due to the stiffness of the stiffeners, the smallest eigenforms and bifurcation factors belong to individual fields. As the stiffeners remain flat when the individual fields are buckled, they can also be considered buckling field edges. Thus, the buckling verification of the individual fields can be performed separately by forming the Navier support conditions from the nodal lines of the stiffeners. As a result, the buckling checks with the resulting bifurcation factors $\alpha_{cr,local}$ can be carried out for the individual fields, considering the possible column-like behavior. Finally, as in Method 1, the buckling verification of the stiffened plate is carried out for the entire field, considering the column-like behavior. In this case, the smallest bifurcation factor $\alpha_{cr,global}$ is calculated, which implies global buckling and, hence, buckling of the stiffeners [5]. This method is more time-consuming due to the separate analyses but often more economical overall.

2.3 Flexible stiffeners

According to the German national annex to EN 1993-1-5, section 4.5.1(3) [1], stiffeners with a related stiffness of $\gamma < 25$ are negligible. However, in [5] it is suggested that the minimum stiffness requirement of stiffeners should generally be observed when designing stiffeners, as otherwise higher load capacities could be achieved than in a load capacity calculation using FEM according to Annex C. In the case of buckling fields with stiffeners of low bending stiffness, the bifurcation load of the entire field is decisive (Figure 2). The verification of the individual panels cannot be carried out separately as in Method 2 because the nodal lines of the stiffeners no longer form a Navier boundary support for the individual panels.

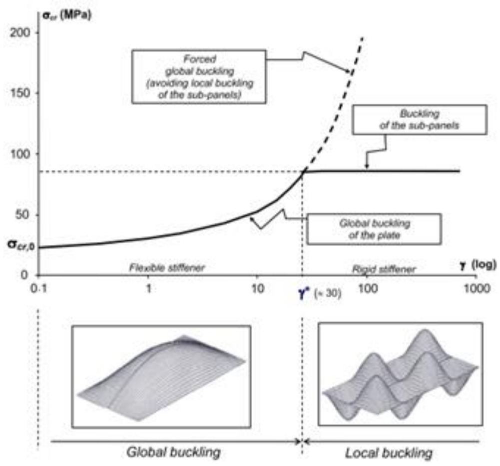

Figure 2 Comparison between global and local buckling [6]

2.4 Underestimation of column-like behavior

In the case of the buckling verification, the column-like behavior must be checked when determining the reduction factor ρ_c. Eurocode provides a formula apparatus for stiffened plates with which the critical buckling stress of the relevant stiffener, including the plate components involved, can be determined relatively accurately. In the case where $\psi<0$, the buckling load of the stiffener with the greatest compressive stress is extrapolated to the buckling load of the whole plate. For unstiffened plates, subsection 4.5.3 clause (2) in [1] gives a stress-independent formula for determining the critical buckling stress, which is obtained by solving the DGL of the linear buckling theory by freeing the longitudinal edges of the plates and applying uniform compressive stress at the transverse edges. The critical buckling stress is higher for plates with a linear stress distribution over the plate height. As a result, the interpolation factor for the column-like behavior is overestimated as it is lower at a higher critical buckling stress. A parameter study is therefore carried out based on an FE model created in the course of this paper for verification according to Appendix C [1], in which the elastic critical buckling stresses and loads of 90 unstiffened buckling fields of different geometry and stress ratios are calculated using a linear buckling analysis and the load capacities are then determined in a non-linear calculation. The calculated critical buckling stresses $\sigma_{cr,p}$ and the column-buckling stresses $\sigma_{cr,c}$ are compared with the handbook formulae from [1] and empirical factors are determined to improve the current formula apparatus. In addition, the resulting reduction factors ρ_c are compared with the calculated load capacities of the FEM. The graphical results of the parameter study are shown in Figure 3 as examples of the slenderness ratio b/t = 150 and stress ratio ψ = 0. The study shows that the increase in critical buckling stress depends on the decreasing stress ratio ψ and the aspect ratio α. These two components are fitted together into an empirical factor using various curve fittings, which results in $3^{0,75\alpha(1-\psi)}$. For a stress ratio ψ = 1, the term results in a value of 1.0 and thus represents the original equation of the critical buckling stress. The new formula is:

$$\sigma_{cr,c}^{neu} = \sigma_{cr,c}^{EN} \cdot 3^{0,75\cdot\alpha\cdot(1-\psi)} = \frac{\pi^2 \cdot E \cdot t^2}{12\cdot(1-\nu^2)\cdot a^2} \cdot 3^{0,75\cdot\alpha\cdot(1-\psi)} \quad (1)$$

The buckling values calculated using formulae from the literature [7] are also significantly higher than those obtained from the FE model at small aspect ratios, further overestimating the load-carrying capacity. It can be shown that the deviations in the stress ratios are due to the empirical term. Therefore, the empirical term has also been adjusted so that the new formula (2) can accurately determine the buckling values.

$$k_\sigma = \left[\alpha + \frac{1}{\alpha}\right]^2 \cdot \frac{1{,}9+0{,}15/\alpha}{\psi+0{,}9+0{,}15/\alpha} \qquad 0 \le \psi \le 1;\ \alpha \le 1 \quad (2)$$

A comparison of the reduction factors shows that the load-bearing capacity calculation and the results of the modified formula are very close. On the other hand, when using the current formulae from EN 1993-1-5, the load capacity is overestimated by up to approximately 10%. As a further analysis for even more accurate factors would be beyond the scope of this study, the identified formula changes were considered to be sufficiently accurate and applied in the following calculations.

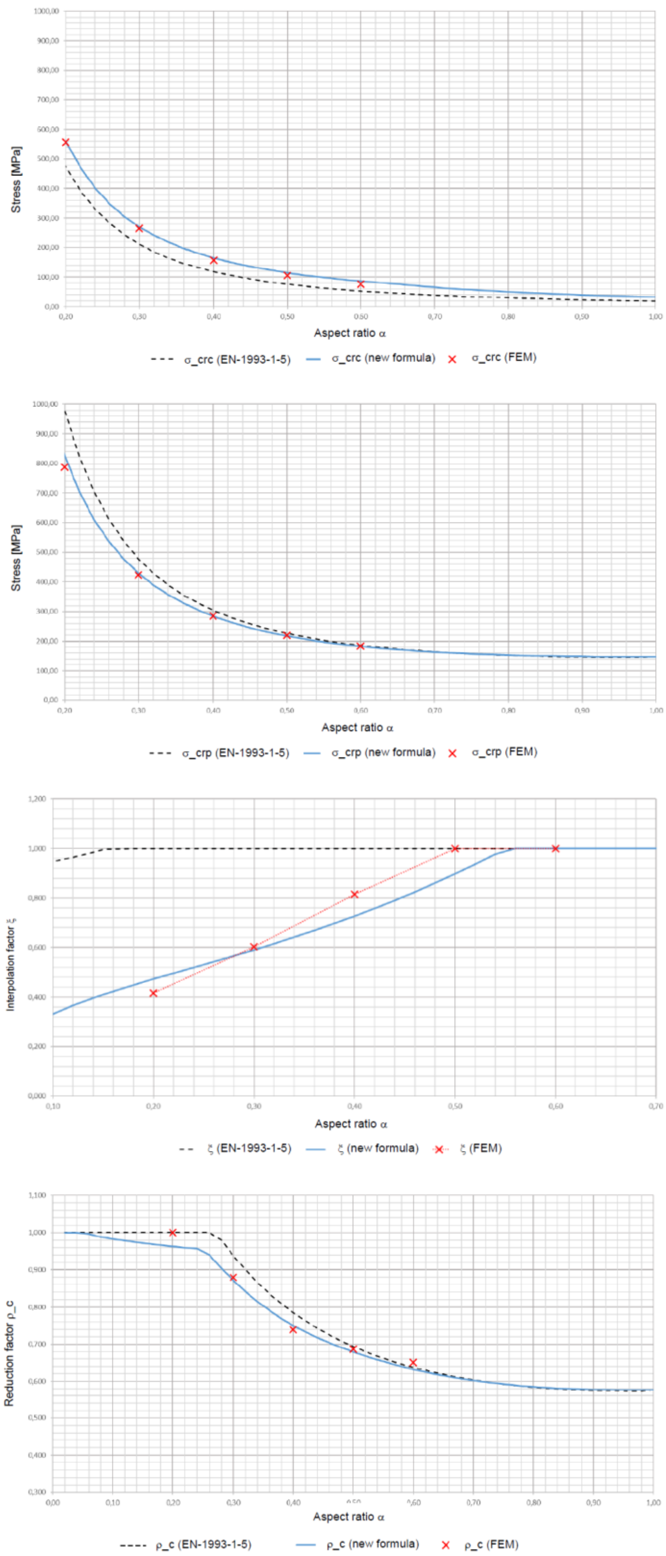

Figure 3 Comparison of current formulas and proposed changes, b/t =150, ψ = 0

2.5 Consideration of shear distortions in the limit state of the load-bearing capacity

Under bending loads, shear distortions occur in flanges due to the uneven application of force. As a result of these shear stresses, the assumed constant stress distribution in the flange is increased at the points of the web connections, resulting in a non-linear (parabolic) stress distribution. By introducing a support width, the influence of these distortions is considered so that the stress in the flange can be considered constant and the structure can continue to be linearly elastic (Figure 4). In paragraph. 3 of [1] formulae for calculating the (elastic) support widths for the service limit state (SLS) are given. In addition, the consideration of shear deformations in the ultimate limit state (ULS) is discussed. The procedures given in [1] for considering shear distortions in the ULS are based on the additional reduction of the contributing areas $A_{c,eff}$ of the flange, which is determined during the verification with the EWM. No information accounts for the shear distortions associated with the RSM. Therefore, a new detection variant is proposed based on buckling detection using the real parabolic stress distribution in the flange. FE programs such as EBPlate [8] allow the modeling of this stress field so that the critical buckling stress is determined considering the shear distortion. As the critical buckling factor is the increased factor of the edge stress, the check is carried out using this edge stress. Therefore, if the effects have been determined using elastic strains, the values do not need to be recalculated. For unstiffened buckling fields, it can be seen that the increase in buckling value, even for buckling fields with small aspect ratios, depends only on the reduction factor and can be expressed by the following formula:

$$k_{\sigma,neu} = k_{\sigma,\psi=1} \cdot \beta^{-1,25} \quad (3)$$

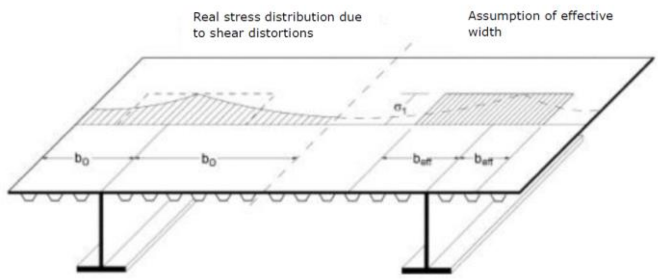

Figure 4 Load-bearing width for thrust distortion [3]

Figure 5 illustrates the options for considering shear deformations and plate buckling together for unstiffened plates with the reduction coefficient β=0.7.

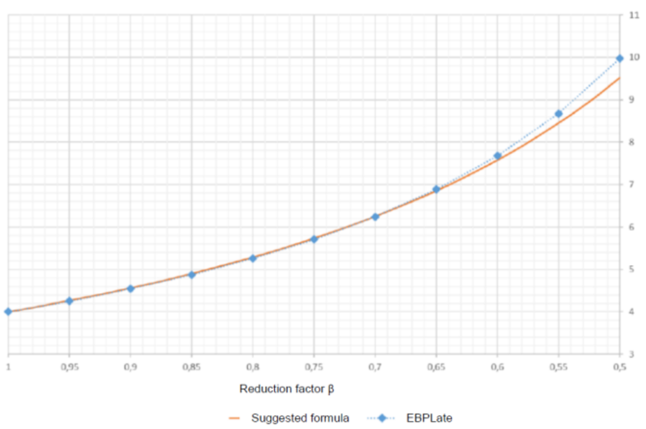

Figure 5 Buckling values k_σ depending on the reduction factor β [3]

3 Development of an automated plate buckling verification

The static calculation of the bridge superstructure is carried out using the SOFiSTiK AG programs [9], where the box girder is modeled as a truss. For the buckling check according to RSM, the individual (un)stiffened plates of the cross-section have to be analyzed, whereby the amplifying factors are determined based on the member stresses calculated with SOFiSTiK in EBPlate [8]. By exporting the data of the required sheet metal geometries and acting stresses from the structural model database, the bifurcation values can be automatically calculated in MS Excel in EBPlate batch mode based on created macros. As a result, the verification is performed automatically in Excel so that the buckling design is directly linked to the structural calculation.

For the automatic verification method to work correctly, the stiffeners must already be modeled in the SOFiSTiK beam model with the correct geometry and position in the cross-section due to the data exports. For example, in the example structure, the lower web stiffeners are modeled even though they are not part of the model (Figure 6, rigth).

In addition to the simple buckling check, an optimization tool has been developed that adjusts the plate thickness of a component in several iteration steps until the stress ratio is 1.0. Although the stiffness and, thus, the internal force curves in the overall structure change due to different plate thicknesses, the tool can be used to predict relatively well the minimum plate thicknesses required for buckling verification so that optimized sheet metal distributions can be achieved in the course of a recalculation of the structure. This makes it possible to optimise the cross-sections of bridge structures with regard to their risk of buckling and to make them more economical with less effort. An automated buckling verification using the EWM is being developed so that the results of the two verification methods for the unstiffened section can be compared. Geometry and stress data are also exported from SOFiSTiK to Excel where the ineffective areas are calculated in tabular form. A recalculation of the cross-sections with the effective areas is then performed in SOFiSTiK (Figure 6, left) and the new stress distribution is calculated for the corresponding effects. Finally, the interaction verification is performed in Excel. As the effective widths are stress dependent, the cross-section values have to be recalculated, which is accelerated by the automated procedure.

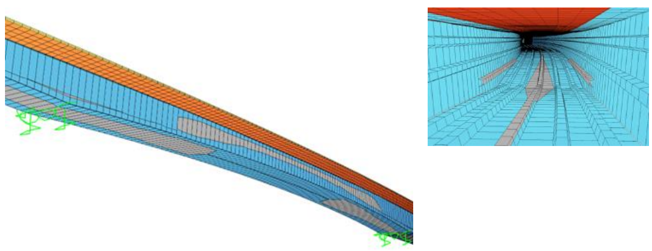

Figure 6 Modelling of the structure (above) and effective widths (below) in SOFiSTiK [9]

4 Calculation and optimisation of the entire structure

Several different cross-sections are studied. They differ in the number of stiffeners (unstiffened or one to five stiffeners) and the thickness of the stiffeners (8 mm and 10 mm). For the different cross-sections, the optimization tool determines the minimum required plate thickness in the whole structure, varying the spacing of the cross frames and, thus, the buckling field lengths from 1.0 m to 5.0 m. The results of these variant comparisons are shown graphically in Figure 7 as an example of the unstiffened cross-section.

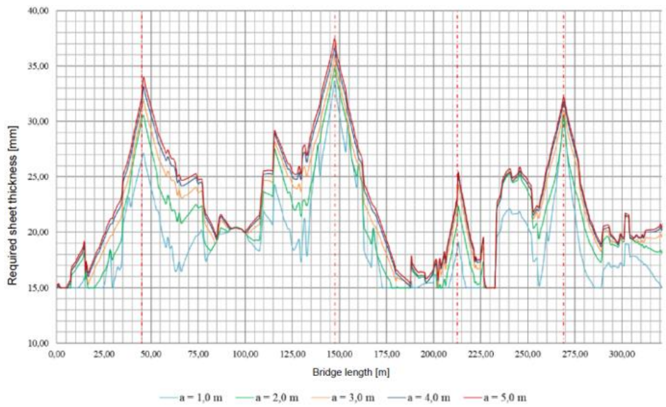

Figure 7 Required plate thickness, unstiffened design variant

Figure 8 shows the selected plate thicknesses of the longitudinally stiffened cross-section design proposal and the final maximum utilization rates.

In addition, it will be investigated how great the effects are on the stiffened structure if design method 2 is selected for the web. For this reason, the longitudinally stiffened design variant is additionally verified according to method 1 (Figure 9). It turns out that design method 1 leads to significantly higher utilization rates, which can be confirmed by the fact that in cross-sectional parts with high related stiffnesses of the longitudinal stiffeners, design method 2 yields significantly higher cost-effectiveness.

5 Conclusions

In this paper, suggestions were made on how reliable and equally economical results can be achieved by additional specifications for using the respective design method depending on the related flexural stiffness of the longitudinal stiffeners. In addition, it makes sense to observe and further elaborate the derived formula to consider the column-like behavior of unstiffened plates since the treated boundary conditions with aspect ratios $\alpha < 1$ and stress ratios $0 < \psi < 1$ can occur in construction practice. The studies carried out on the joint consideration of shear distortions and plate buckling could also be considered in future

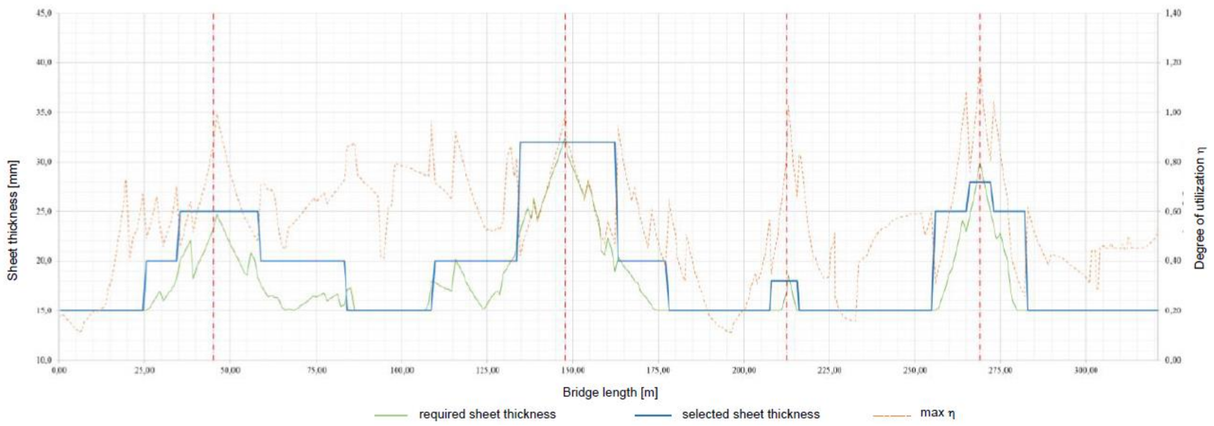

Figure 8 Selected (optimised) and required web plate thickness, longitudinally stiffened design variant

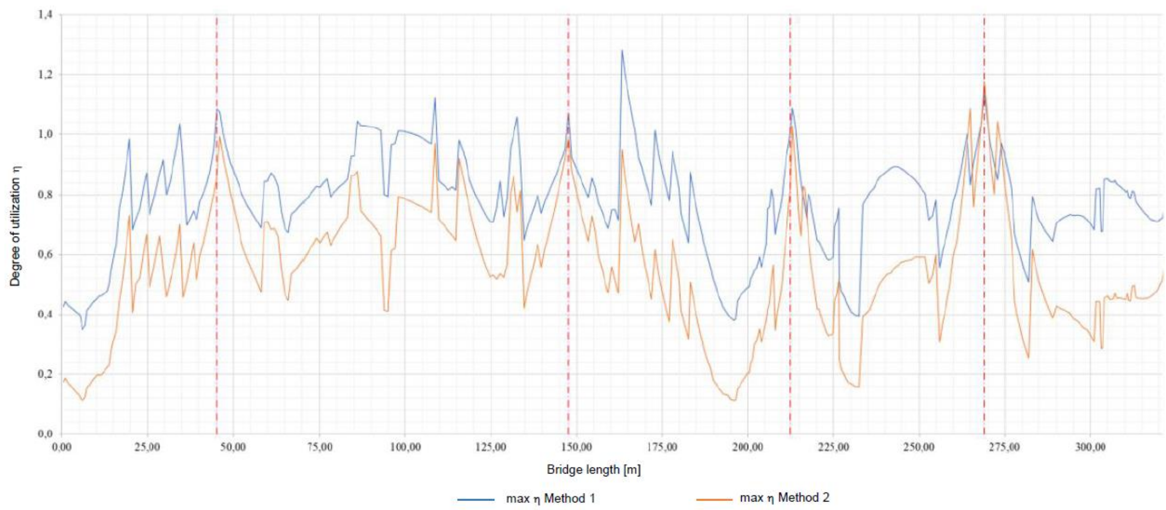

Figure 9 Comparison of Method 1 and Method 2 for the stiffened web plate

changes to the standard. In optimizing the example bridge, the performance of the developed verification tool could be determined. An unstiffened variant should also be considered, as it has further advantages and can be more economical overall with lower cost differences. Further results can be found in [11].

References

[1] DIN EN 1993-1-5 (2010) Eurocode 3: *Bemessung und Konstruktion von Stahlbauten Teil 1-5: Plattenförmige Bauteile*; Deutsche Fassung EN 1993-1-5:2006 + AC:2009 + A1:2017 + A2:2019 + Nationaler Anhang, 2019.

[2] „Ingenieurbüro Grassl GmbH Beratende Ingenieure im Bauwesen," [Online]. https://www.grassl ing.de/.

[3] Braun, B.; Kuhlmann, U. (2009) *Bemessung und Konstruktion von aus Blechen zusammengesetzten Bauteilen nach DIN EN 1993-1-5.* Stahlbau-Kalender, Berlin: Ernst & Sohn GmbH & Co. KG, pp. 382-451.

[4] DIN EN 1993-2 (2010) Eurocode 3: *Bemessung und Konstruktion von Stahlbauten - Teil 2: Stahlbrücken*; Deutsche Fassung EN 1993-2:2006 + AC:2009 + Nationaler Anhang, 2010.

[5] Kuhlmann, U.; Schmidt-Rasche, C.; Frickel, J.; Pourostad, V. (2017) *Untersuchungen zum Beulnachweinach DIN EN 1993-1-5.* Bericht zum Forschungsprojekt FE 89.0313/2015. Bd. Heft B 140. Berichte der Bundesanstalt für Straßenwesen Brücken- und Ingenieurbau, Bremen: Fachverlag NW.

[6] Galéa, Y.; Martin, P.-O (2010) *Longitudinally stiffened plates in Eurocode 3: Calculation of the global critical buckling stress*, Journal of constructional steel research 66, Elsevier Ltd. pp. 1345-1353.

[7] Kindmann, R. (2008) *Stahlbau: Teil 2: Stabilität und Theorie II. Ordnung*, 4. Auflage, Berlin: Ernst & Sohn.

[8] EBPlate (2013): *Im Rahmen des COMBRI-Forschungsprojekts entwickelte Software zur Ermittlung der elastischen kritischen Beulspannungen.* CTICM https://www.cticm.com/logiciel/ebplate/.

[9] „SOFiSTiK AG" 2020. https://www.sofistik.de/.

[10] Maier, N.; Pourostad, V.; Ndogmo, J.; Mensinger, M.; Kuhlmann, U. (2019) *Abschlussbericht Talbrücke Thulba: Beuluntersuchungen*. TU München und Universität Stuttgart.

[11] Mertens, Th. (2020): *Entwicklung einer automatisierten Nachweisführung gegen Plattenbeulen für Stahlverbundbrücken mit Diskussion der Normung und Vergleichsrechnungen mit der FEM*. Master's thesis. Lehrstuhl für Metallbau der TU München.